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THE USE OF STEEL FOR BRIDGES.

By THEODORE COOPER, C. E., Member of the Society.

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The substitution of steel for iron for bridges has become a prominent feature of discussion among engineers and bridge builders.

Some are still skeptical upon the propriety of using it; others are sanguine enough to believe that all that is now necessary, in reference to the introduction of steel, is to use it.

The problem, however, is not so simple, although it is no doubt true that the more we use it the better will we know how to use it.

Steel must enter upon a struggle for precedence with iron, somewhat similar to that which iron has been undergoing, with reference to wood for the past forty years, and will undoubtedly come out in the end as victorious.

It will be instructive and interesting to look back upon the past development and history of American bridge building.

The first characteristic American bridges were built almost exclusively of wood—iron only entering into some of them in the form of straps and bolts. Wood was cheap and ready at hand; it was easily worked into th

required shapes by the skilled labor found at all points; and all the work could be performed upon the building site.

About 1840 an advance was made toward a further use of iron and a simplification of the labor of framing the wooden portion by the introduction of plain iron rods with nuts at each end, for the tension members of the web system.

This stage brought us to the perfection of our styles of wooden bridges—the Howe bridge—a bridge which still disputes with iron the right of existence in many parts of our country, due to its low first cost and facility of construction.

Its merits of cheapness and readiness of construction would have prevented for some years a further advance towards our present styles of wrought iron bridges if the factors of rapid decay and liability to burn had not entered into the question.

Iron, with reference to its application in any other form than as simple rods, was a new material. The questions of its first cost, the reliance which could be placed upon it for any new adaptation, the forms of its details, the mode of working it into proper shapes and the manner of obtaining accurate dimensions were all to be met.

Gradually and slowly, in the hands of the able engineers and builders who had undertaken this specialty, our present well developed and reliable styles of wrought iron bridges were accomplished facts.

Only those who have been engaged in the building of iron bridges from the beginning to the present day could give a full and correct account of all the thought, anxiety and patient experiment gone through with to fully develop the present forms in use in wrought iron construction.

At first cast iron was almost entirely used for compressive members and for making the connections of the several members. The old wooden forms were closely adhered to, not only in reference to the general forms of the trusses, but also in the mode of connecting the parts.

The tension members were made of wrought iron, but it was some years before they assumed our present form of eye bars. In some of the earlier bridges the lower chord was built up very much like the chords of the wooden bridges—it being formed of long flat bars welded in one length and combined together like a wooden chord—the connections to the posts and web rods being by means of angle blocks of cast iron, which connected to the lower chord by lugs fitting into notches cut into these bars.

In others the lower chord was made of long loops, link shaped, made by bending and welding together the ends of a long square bar. These connected to trunnions cast upon the foot of the posts. The use of wrought iron pins was afterwards introduced.

In others a nearer approximation to our present form of eye bars was made by looping over the ends of a square bar and welding these ends to the body of the bar; thus making what is still used in places, and is called a bar with looped eyes.

The room taken up on a pin by these square bars created the demand for flat bars which could be packed closer together. This necessity led to the development of a method of making a flat eye bar such as is now used.

The cast iron gradually, but stubbornly, gave way to improved forms of wrought iron, and now we have true wrought iron bridges.

Their construction has demanded large investments in workshops with special plant, created new demands upon the capacity of rolling mills and developed a special class of workmen. Iron bridge building is now a special branch of engineering science. It is not a knowledge of strains alone that is necessary to enable one to design successfully a wrought iron or steel structure. Where such structures are thrown open for competition the structure which costs the least and satisfies the requirements, as regards the quality of the material, ability to meet the strains, method of details and general workmanship, will be the best. The practical designer must have a large experience and a clear judgment of the influence of the several factors which enter into the problem. The theoretical solution of the most economical proportions will be vitiated by the necessity of taking into account the following factors:

1st. The use of such forms and sizes of material as can be obtained in the market.

2d. The possibility of combining these forms into shapes and proportions suitable to the purpose and to the strains they are to resist.

3d. The relative prices of different sizes and forms.

4th. The relative cost of manufacturing these into the required shapes.

5th. The limiting sizes to be used regardless of the smallness of the strains.

6th. The cost of transporting and erecting different designs.

7th. The liability of long and light pieces to be distorted and injured during transportation and erection.

8th. The relative cost of suitably bracing structures of different designs and proportions.

9th. The relative cost of making the connections of the various parts.

10th. The objectionable looseness of a structure carried to a theoretical lightness.

Having thus briefly considered the circumstances governing the introduction and use of iron bridges let us consider what are the prospects of the substitution of steel for iron.

The word "steel" is now used to represent a variety of alloys of iron and carbon modified to a more or less extent by the unavoidable or intentional addition of certain other metals or metalloids, as phosphorus, sulphur, silicon, manganese, chromium, tungsten, titanium, &c., the first four being nearly always present, modifying or intensifying the hardening action of the carbon.

As the engineer applies the finished material, iron or steel, to perform certain duties, whether it be as a bridge, ship, boiler, tool, or machine, where the material has to resist the forces acting upon or through it by its physical attributes, it should be his aim and duty to determine what *physical* qualifications the material should possess to best perform its desired work.

The chemical investigation of the properties of various grades of steel belongs to the chemist and producer of the material. In either iron or steel it is the duty of the Engineer and worker of the steel or iron to specify what are the characteristics, from their field of experience, required, and in no manner to hamper the producer of the material by specifying the grade of ore, kind of scrap, or mode of working. By different methods of working, whether it be in the puddling or smelting processes, or in the manner of hammering and rolling, different results will be obtained from the same ores or same scrap, and equally good material for definite purposes can be made, for the same reason, from entirely different raw material. When any chemical element is present in the product, or any condition of the process is known to give uncertain results, some being good and others bad, the engineer is perfectly justified in demanding, under such circumstances, a more onerous and expensive series of tests or guarantees in order to satisfy himself of the uniformity and reliability of the material.

Neither has the engineer any right to brand any process of manufac-

ture by specifying that only such a process or processes will be allowed. If it is known that steel or iron made by any one or more processes has objectionable characteristics, the specification should be drawn to exclude material with *such characteristics*, thus leaving all manufacturers to improve or vary their product as best they can.

Open competition, upon a clear and well defined specification, will soon develop the best mode of making and the proper chemical composition of the material.

Moreover, chemists and experts in the manufacture of steel are themselves largely at variance upon the influences exerted by fixed proportions of the chemical constituents.

With this explanation, we will confine our remarks to the physical characteristics of steel. The most important of these are the tensile strength and the ductility. These *largely* represent its suitableness for engineering purposes. The range of the tensile strength in steel for practical purposes extends from about that of the softest iron (45 000 lbs.) to four or five times as great. Its ductility (when measured upon a standard specimen eight inches between points of reference) ranges from about 50 per cent to nothing or extreme brittleness.

Within this great range of qualifications an engineer ought to be able to find steel suitable for any structural or mechanical purpose, and will be able to do so when our experience has been sufficiently extended to enable us to determine the kind best suited to our purpose.

In making such a selection he must be governed not only by the *character of duty* which the material must be *adapted to do*, but must also take into consideration its *ability* to *undergo the processes of manufacture* in preparation for its actual use.

In selecting steel for bridge purposes, such a quality of material must be chosen as can stand the processes of rolling into the various shapes demanded; can be bent, cut, punched or drilled, and worked into finished members with the least injury to its structural requirements, and can be handled through all its processes from the ingot till it is a finished bridge without more than ordinary care, (parts of iron structures have frequently to pass through severer tests during the processes of manufacture, transportation, and erection than after they are doing duty as part of a completed work).

Ductility being that characteristic which indicates the relative ability of the metal to flow under a strain exceeding the elastic stress, is there-

fore the property which governs the capability of adjustment to local or internal stresses produced by the various mechanical processes. The ductility is, however, only gained by a sacrifice of the high tensile resistances of the harder steels. In other words, the *full cohesive action* of the material cannot be developed as long as the molecules have the power to move among themselves at a low strain. That this motion of the molecules is governed by a frictional resistance seems very plausible from the following facts:

1. The existence of an elastic limit or point of strain up to which the flow or molecular action does not occur.
2. The great development of heat in the metal during such flow.
3. The increased limit of elasticity and ductility imparted to iron and steel by the "kneading" it receives from the processes of rolling and hammering.

But apart from any theory upon the subject, the fact is well known that great ductility is accompanied by a low tensile resistance, and a high tensile resistance by a diminution or total absence of ductility.

The usual measure for the ductility is the amount of elongation, in percentage, of a certain length of specimen under its ultimate resistance.

As specimens of the same material, in different forms, will give different results, and as the reduction of the specimen is much more rapid in the immediate vicinity of the fracture, it is absolutely necessary to have all comparative tests made upon the same form and size of specimen. It is impossible to make any use, for general deductions, of a large percentage of the tests heretofore made upon iron and steel, from the fact that, even when the details *are* all recorded, the varying sizes and shapes of specimens prevent any comparison. Experimenters all over the globe have recognized so well the importance of this fact, that an effort is being made to establish by mutual acceptance a uniform size of specimen.

A large number of experimenters, and most of the governmental boards in Europe, have accepted or approximated to 8 inches as the length upon small specimens for estimating the elongation. For round specimens a diameter of about ¾-inch, and in flat specimens a width from 1½ to 2½ inches, (the former would seem to be a nearer approach to a comparative test with the round specimens than the latter).

From the result of experience, and experiment made upon steel for use in boilers and ships, where the mechanical processes and strain actions are very similar to those in bridges, it has been found expedient to

limit the grade of steel to that which has the following physical characteristics : a tensile strength between 60 000 and 75 000 lbs. per square inch, and an elongation upon a specimen of 8 inches of 25 to 15 per cent., the more conservative tendency being towards the lower tensile strength. The British Admiralty requirements being as follows : "Strips cut lengthwise or crosswise to have an ultimate tensile strength of not less than 26 tons (58 240 lbs.), and not exceeding 30 tons (67 200 lbs.) per square inch of section, with an elongation of 20 per cent. in a length of 8 inches, the beam, angle, bulb, and bar steel to stand such forge tests, both hot and cold, as may be sufficient, in the opinion of the receiving officer, to prove soundness of material and fitness for the service."

"Strips cut crosswise or lengthwise 1½" wide, heated uniformly to a low cherry red, and cooled in water of 82° Fahr.; must stand bending in a press to a curve of which the inner radius is one and a half times the thickness of the steel tested.

The strips are all to be cut in a planing machine, and to have the sharp edges taken off.

The ductility of every plate, beam, angle, &c., is to be ascertained by the application of one or both of these tests to the shearings, or by bending them cold by the hammer.

All steel to be free from lamination and injurious surface defects." The French requirements are almost identical.

As bridges are composed of various forms of members which have to perform entirely different duties and which pass through mechanical operations of different kinds, let us consider them more in detail.

American bridges are generally built up from the following individual members, most, if not all the mechanical work upon them being done in the shop.

1st. Chord and web eye-bars; round, square or flat bars with a head at each end, formed by some process of forging.

2nd. Lateral, diagonal and counter rods ; bars of a lighter section than the above ; sometimes with a forged eye at each end and a screw adjustment at some intermediate part of the rod. Sometimes without eyes, and having a screw thread on each end. The portion of these rods which has a thread upon it, is upset to a larger diameter than the body of the bar (generally ¼ inch) for a length of 6 to 8 inches.

3d. Floor-beam hangers. These are of various patterns, but most usually are bent loops with screw threads cut upon the upset ends.

4th. Pins.

5th. Lateral struts.

6th. Posts.

7th. Top chord sections.

The last three being columns formed by riveting together various rolled forms; plates, angles, channels, I beams, and some special post forms. Some are square ended, others pin-connected.

8th. Floor-beams and stringers. These consist either of rolled beams, riveted plate girders, or occasionally of latticed or trussed girders.

We will now examine these separately, considering their respective strain duties and the several mechanical manipulations which they must undergo, to see what, according to our *present* knowledge, is the character of steel best suited to their purpose.

1. Chord and web eye-bars. These are tension members, and are of uniform section throughout their body, and have at each end a gradual enlargement which forms the eye for connection to the pins. The work put upon these bars is in forming the eye and boring the pin-hole—the first operation being the only one which can do an injury to the material. Whether a head can be successfully and certainly formed by either the process of upsetting or welding, without injury to the material, is so dependent upon the character of the plant used and upon the quality of the steel, that it remains for *future* tests to satisfy us in regard to this matter.

According to our present knowledge, the process of Mr. Kloman of Pittsburgh, of rolling a bar down in the centre so as to leave a portion at each end of greater thickness from which the head can be hammered or pressed out, seems the most satisfactory. It would, however, be very rash to assert that no other process *may be* developed.

If the eye can be formed upon these bars without injury to the character of the material, and if the length of the bars is not so great as to risk their injury by bending during the process of working, transportation or erection, a comparatively high limit of strength *might* be adopted for them, say 80 000 to 85 000 pounds per square inch. with an elongation in eight inches of never less than fifteen per cent. As the steel to fill this condition would require to be very well worked, it may, for the present be impracticable to obtain so high an ultimate strength with that minimum of elongation, and this elongation is the important factor to be insisted upon whatever the tensile strength may be.

2. Lateral, diagonal and counter rods. These being more liable to be bent in handling, and being subject to a more sudden strain from rapidly passing trains, should be of a more ductile quality than the above. A similar material to the above can be used, as the greater amount of rolling required would increase the ductility, but the elongation should be limited to at least twenty per cent. As steel will not stand long under the sharp grooves of the ordinary forms of screw-threads a change in the form of threads should be made. The bottoms of the grooves should be rounded instead of the truncated V form as now used.

3. Floor-beam hangers. As these receive the direct action of sudden loads, and are so vital to the safety of passing trains, they should also be made of a very ductile quality of steel.

4. Pins. The severest test that these must undergo being from a bending strain, and having to pass through no injurious manipulations, a hard steel could be adopted.

5, 6, 7. Compression members. The natural conclusion of those familiar with the characteristics of different steels, would be that for these members a high quality of steel is most essential; but, as before stated, the mechanical manipulations through which they must pass have also to be considered. The shearing, bending, straightening, punching, planing, &c., through which these members must pass are operations of the severest kind for steel, and could only be justified upon the softest steels. Moreover, frequent cases have occurred (one came recently within my knowledge) where, from derailment of a car, posts have been bent out of shape, and the bridge saved from destruction with its accompanying loss of life, by the toughness of the iron posts. Now, a steel post, whether drilled or punched, would not be able to stand such a test unless it was of a material equally ductile.

8. Rolled beams and plate girders. The material for the rolled beams will be largely governed by the process of rolling into shape. For plate girders the same remarks as have been expressed for compression members would be still more applicable as parts of them are subject to tensile strains; and steel which has been reduced to irregular sections by numerous holes, whether these holes have been drilled or punched, will not have the same power to resist repeated strains that a uniformly prismatic bar would have.

The necessity of annealing all pieces of steel, whether soft or hard, that have been unequally or locally heated, or locally strained in any

manner by the several processes, has not been mentioned in our consideration of the quality of the steel to be used for the several members of a bridge structure; vital as this process is to a successful result, it will not restore steel that has been incipiently ruptured by careless forging or other mechanical work. The only benefit of annealing is to relax undue local strains. If the steel is so hard as to be cracked at the edges of the holes or shear cuts, or by unequal cooling during the process of forging or rolling, or by cold hammering, however imperceptibly, the strength of the metal cannot be restored by any after process of manipulation. Hence, it is absolutely necessary to keep the quality of the metal down to the point where it cannot be thus injured by the particular processes to be adopted in the manufacture. That after processes will be developed by which these different members can be formed without such severe manipulation is certain, but for the present transition stage we must consider the methods and plant we are now prepared to use.

We have endeavored, in the preceding remarks, to make clear the importance and necessity of requiring a ductile metal regardless of what its tensile strength may be. This ductility must be that of the actual rolled material, and not that of the ingot metal, or samples of the ingot metal worked in a different manner from the material to be used. The condition of the actual forms cannot be judged in any other manner than by taking the actual pieces or proper specimens from them.

The amount of tensile strength that can be obtained in connection with a specified percentage of elongation is dependent upon two factors: one is the chemical composition and the other the amount of work put upon the metal during the forging or rolling. The first, as before explained, is only of importance to the user of the material, as it may impart new *physical attributes*, and these are the means by which he should determine its appropriateness for any particular purpose. If it will not harden or become brittle or treacherous under any condition of heat or work to which it will have to submit, it is unimportant for his purpose what the proportion of carbon, phosphorus or other ingredient may be. Even if he knew the accurate composition, he is still compelled to depend upon his physical tests to be assured of its quality.

The second factor, or amount of work put upon the metal, will be governed by the capacity of the plant by which it is to be worked. Therefore, we cannot expect so large a tensile strength in the heavy

sections as in the smaller ones. Competition will soon develop the capabilities of our manufacturers of steel, when a sufficient demand has been created for a steel with definite characteristics suitable for bridge purposes.

In the meantime, we do not think any bridge builder or engineer is justified in requiring a steel of high tensile strength, unless he has first satisfied himself, by a thorough series of tests, that the steel can be obtained in suitable forms and will stand the required manipulations without injury.

The cost of the St. Louis Bridge was largely increased by a demand for forms and material beyond the capacity of the period; new plant had to be created, and much time lost by experiments to obtain the requisite parts of the structure. Special cases may occur where the importance of the work will justify the loss of time and the additional expense created by excessive demands, but generally the question of cost will restrict the requirements to the known possibilities. The following requirements for bridge steel should, in our opinion, be the *maximum* as to tensile strength and *minimum* as to elongation demanded, until increased experience proves the safety of changing them. Tests to be made upon the standard form of specimen.

For plates, angles, channels and other shapes an ultimate strength between 65 000 and 70 000 per square inch, elongation not less than 20 per cent. in 8 inches; limit of elasticity above 35 000 pounds per square inch. For small bars and rods, an ultimate resistance between 75 000 and 85 000; elongation not less than 20 per cent. in 8 inches; limit of elasticity above 40 000 pounds per square inch. For large flat bars, an ultimate resistance between 70 000 and 80 000; elongation not less than 15 per cent. in 8 inches; limit of elasticity above 38 000 pounds per square inch. In addition, the steel must be satisfactory as to its hardening tendency, bending tests, etc., with such other practical conditions as may ensure a certain and reliable material for the required purpose.

In this connection it will be well to draw attention to one or two points which are frequently overlooked:

1st. The importance of a more accurate measure of the tensile elongation in reference to the rapidity of the applied strain. It is well known that the tensile strength and elongation can be made to vary by applying the strain more or less rapidly; and partly due to this cause different machines or different operators of the same machine may obtain different results upon the same material.

2d. The necessity of determining the ductility of the metal at different temperatures. Certain mild steels, which can be bent cold or hot, break readily by percussive action at a medium temperature (600°).

3d. The desirability of testing all steel by impact or a falling weight. It has been found that certain steels, showing the same results under a tensile test, give varying resistances to impact, which would seem to show the effect also of a variation in the rapidity of the applied strain. At a slow velocity of strain the two metals may flow alike, but at a higher velocity their results are different.

What is the increase in the working strain that can be adopted for this class of steel over that used for the same members in iron?

Bearing in mind that the above tensile requirements apply to specimens, and that upon long pieces such as would be used in practice, both the ultimate and elastic stresses would be considerably lower; and that the reduced weight of the structure would increase the effects of impacts and vibrations of rapidly moving trains, or from derailment of part of a train; we would not deem it advisable to increase our customary working strains used for iron bridges more than 50 per cent.

What kind of steel, in reference to the make, will be most suitable for bridges? This question must be decided by the relative cost of such material as will fill the requirement.

The three processes of making steel, Crucible, Bessemer and Open Hearth can all undoubtedly fill these requirements; their relative cost must, therefore, decide the question. This relative cost does not consist solely upon what it may cost each process to make a ton of steel (in general), but what it will cost them to make a specified amount of a particular steel, and to get rid of such ingots as will not suit the purpose. This will largely bring the question down to the comparative certainty by any one process or establishment of obtaining the exact steel required with the least waste or surplus of unprofitable material.

The additional cost of smelting would apparently rule out the crucible steel. For the other processes it would be reduced to a competition between establishments.

What effect will the substitution of steel for iron have upon the proportions for economy, and upon the form of the several members and their detail parts? That there will be a change in proportions best suited for economy is very probable, but it can only be solved by practical competition between different designs, as it has been in iron; it

appears likely, that relatively lower heights of spans will be adopted for moderate sized spans, at least—owing to the necessity of avoiding too thin sections of metal.

As to the latter portion of the above question, the form of members and their details, undoubtedly, a change will be required. The best form of eye and proper size of pin-hole in eye-bars, the best and most economical method of riveting together the parts of members, the best form of compression members, etc., can only be definitely determined by future experiments. Experience already obtained upon steel teaches us the necessity of avoiding all local strains; not only those due to mechanical work but also those due to local attachments. Greater care must, therefore, be taken to transfer a stress uniformly over the whole section of a member. This would point towards the use of a greater number of small rivets instead of a few large ones for making local attachments to the webs of plate girders, sides of posts and chords, &c.

Sharp re-entrant angles must be avoided by the introduction of a large fillet.

Time, with its increasing experience and the ingenuity of designers, will, undoubtedly, develop new and proper forms for the several parts so as to avoid all objectionable features and expensive processes.

It is to be hoped that we are to receive from the Chief Engineer of the Glasgow Bridge a report of the experiments made and experience gained during its construction. A full account of the condition of all parts of the wreck of the fallen span would give us much information. It would be very desirable to have the doubtfulness of steel, under certain conditions, disproved if possible.

To what extent can steel be economically substituted for iron in different sized structures and in different parts of the structure?

The proportionally great effect of oxydation upon very thin or small sized members, and their small resistance to accidental blows, has shown the necessity of limiting, in iron, the thickness of sheets to a minimum of about one-fourth of an inch, and the diameter of the smallest rod to about one inch. Similar reasons would justify the adoption of the same sizes in steel.

The practical difficulties of the rolling process will also establish limiting thickness for channels, angles, beams and other shapes, in steel as it has in iron. And it is not probable that these shapes in steel can be rolled very much, if any, thinner than in iron, and certainly not

thinner in proportion to the relative strength of the two materials. In any sections, therefore, which approximate to these minimum sizes, there will be no gain in the use of steel, unless its price is near that of iron.

Upon the supposition that the cost of steel is proportionally equal or less than the cost of iron, considering their relative working strains, steel could be substituted for the tension members of the trusses in all but exceedingly small spans.

In the lateral system, steel rods could only be used in bridges over 150 feet.

Whether the pins in any sized span should be changed to steel, would depend largely upon the methods of transferring the strains to and from the other members, it being questionable whether large iron pins would not answer this purpose better than smaller steel ones.

In compression members, as we must retain relatively the same diameters as in iron, if we are to increase the strain proportionally to the strength of steel over iron, we cannot reduce the weight of lateral struts in spans below 150 feet. Neither can we reduce much the weight of the intermediate posts and lighter top-chord sections in spans below 150 feet.

The use of rolled beams will be limited by the fact that they deflect under equal strains nearly the same as iron and cannot be rolled much thinner.

The reduction in built beams will depend, likewise, upon the span and the limited sizes of the parts. The thickness of the web plates cannot be proportionally reduced, as their thickness will be dependent upon their power to resist buckling.

From such detailed considerations it is evident that there will be no economy in the use of all steel in spans of ordinary sizes (single track bridges up to 150 or 175 feet) as long as steel is dearer than iron. At what exact points the use of all steel will be the most economical, can only be determined by comparative estimates upon particular plans. When the length of spans becomes greater, an appreciable saving in weight to be carried increases the economy. In spans from 200 to 300 feet long, this will amount to a saving of about 5 to 12 per cent.

Relative cost of manufacturing steel and iron bridges.—This can only be answered by a more extended experience than we now possess, and in the future will largely depend upon the development of new plant and new methods of manipulation. But for the present, considering the ad-

ditional care required in all the processes and the necessity of substituting temporarily processes which are more costly than those heretofore used in iron structures, the cost of manufacturing steel structures must be higher than for iron ones.

As a believer in the great adaptability of steel for mechanical and structural purposes, only limited by its relative cost and a proper knowledge of the manner of working and using it, the writer has endeavored to draw attention to such facts as would seem to govern the successful introduction of the use of steel.

As the first successful iron bridge builders were those who were able to throw aside the traditions and practice of the users of wood and adapt their designs and processes to the capabilities of the metal, iron—so must the successful builders in steel be those who can accept the fact that the new metal, structural steel, requires a like sacrifice of old traditions and practices, a development of new plant and processes, and the education of a new class of metal workers.

DISCUSSION

On the Paper, "The Use of Steel for Bridges," at the Meeting of the Society, September 17th, 1879.

By T. C. Clarke, Charles Macdonald, O. Chanute, D. Torrey and Theodore Cooper.

T. C. Clarke.—The Society is much indebted to Mr. Cooper for his exceedingly thoughtful and suggestive paper on the use of steel, which has covered the ground so completely that there is not much left for any one else to say.

I would, however, remark, that I have just received from C. O. Gleim (Member Am. Soc. C. E,), Engineer of the Central Bureau of Rhenish Railways, Cologne, a very interesting letter, from which I quote the following statement as to the abandonment of steel in Holland for riveted girders:

"I do not know whether you have heard of the recent experience of Dutch engineers in regard to the use of steel in bridge building, which has led to its entire abandonment in Holland. It is a well known fact that the Kuilenberg bridge has its floor system constructed of steel; and since its erection, the same system has been adopted for most of the large

iron bridges built in Holland. The only bridge that I have heard of with the main girders constructed of steel, is a plate-girder bridge, carrying a number of railroad tracks of the Amsterdam Central Station across a canal; and it was on this bridge that their steel girders first came to grief. After making a series of experiments, they concluded to give up the use of steel altogether on their government bridges. There were two large bridges, across the Rhine at Arnhem, and across the Waal at Nymegen, in course of erection at the time; and a Dutch engineer wrote to me last summer, that, on the strength of those experiments, the steel floor-beams and track-stringers of the Arnhem bridge were taken out again; and that in the case of the Nymegen bridge, where it was too late to exchange them for iron beams, although the bridge was not opened for traffic, they resorted to the rather doubtful remedy of strengthening them by the addition of new chord-plates. I have unfortunately mislaid the data I have from another engineer in regard to the experiments referred to. But the main fact was, that they built up riveted beams, of their floor-beam section, out of plates and angles that had previously been subjected to severe tests of their tensile strength; and that on loading these beams transversely, they were found to give way under a computed strain only a fraction (down to one-third, I think,) of what they had previously borne. One of the angles would suddenly snap, while the rest of the section continued to bear the transverse strain."

This illustrates clearly the necessity pointed out by Mr. Cooper, of so treating the steel as not to injure it in manufacture. No maker of steel has yet been able to overcome the danger of steel suddenly giving way when nicked or cut into. These Dutch girders were probably injured in punching or drilling, or in riveting.

I would say that, taking into account our ignorance of the ultimate strength of a framed structure, as a whole, as distinguished from the strength of its parts, I would not be willing, in the present state of our knowledge to strain steel bridges even fifty per centum more than iron, which was the maximum recommended by Mr. Cooper.

C. Macdonald.—I have seen some of the steel eye bars that were used in the Glasgow bridge during their construction at the works of Andrew Kloman, Esq., Pittsburgh. They appeared much smoother and more regular than the best wrought iron eye bars ; and from the method employed by Mr. Kloman in forming the eye, the difficulties attending the upsetting or die forging operations seem to be eliminated.

I have also seen photographs of the eye bars that had been taken out of the wreck of the Glasgow span, which was carried away by the ice during the process of erection; they were bent and twisted in every conceivable direction. Nevertheless, they were sent back to the shop,

reheated and straightened, and put back into the bridge. General Sooy Smith, Member of the Society, engineer of the bridge, informs me that only four eye bars were broken in the fall, although they were not so fortunate with the compression members.

When it is remembered that in this accident upwards of 250 tons of finished steel were precipitated from a height of 80 feet into the river, with so little injury to the material, it at least furnishes some evidence of the possibilities of steel as a material for bridge construction.

Referring to the question of ductility, it would be well, perhaps, to consider whether it is necessary that, in general, a low grade of steel should be selected for structural purposes, in order to insure a high degree of ductility.

I find that many makers of steel, who are themselves scientific observers, do not hesitate to use high steels to resist shock and vibrations. Mr. William Metcalf, Member of the Society, an eminent steel manufacturer of Pittsburgh, informs me that he finds cast steel, having as high as $\frac{9}{10}$ carbon, gives the best results in the vibrating parts of agricultural machinery; and at his own works he has used a similar grade of steel for steam-hammer rods, where the shocks are certainly of the most violent character; yet they have outlasted those made from more ductile material.

From Kirkaldy's experiments on the Fagersta steels, it would appear that at $\frac{9}{10}$ carbon a maximum tensile resistance is reached, giving an average of 63 000 elastic stress and 106 000 ultimate; while the ductility, as measured by the contraction of area at fracture was only $\frac{1}{10}$ as great as in the specimen having $\frac{3}{10}$ carbon.

In such portions of a bridge as, for instance, the bottom chords, where great accuracy of workmanship may be relied upon, with our present appliances, would it not be well to take advantage of this superior tenacity of the high steels, by using them at a strain approaching 20 000 per square inch, rather than the figure mentioned in the paper?

For compression members we are not so sure of the results of the operations in the shop, such as punching, riveting, &c. Hence it would be more prudent to utilize the lower grades until further information is obtained.

O. Chanute.—It is not improbable that some of those who look upon steel as the coming material for bridges shall find themselves rather disappointed at the results arrived at in Mr. Cooper's paper, in

this, that they do not indicate more definitely the economy and advantages which may be expected to result in bridge construction from the substitution of steel for iron.

Such a first impression would, however, be erroneous. It can scarcely be expected that at the very threshold of the introduction of steel, and in the absence of definite information concerning the adaptability and homogeneity of the material, we can reach any very positive conclusions.

Mr. Cooper's paper is, from the very nature of things, in the character of a preliminary inquiry, and the thanks of the Society are due him for furnishing this first nucleus, around which other contributions may crystallize. It now remains for the individual members to endeavor to agree upon specifications for the use of steel in bridges, and to point out what further experiments are needed in order to gain an adequate knowledge of the capacities of this metal.

I quite agree with the author, that the engineer who proposes to use steel should chiefly concern himself with the physical characteristics of the product, and should not attempt to specify to the manufacturer either its chemical constituents or its manipulation. For a while, however, until we know more about it, it is probable that the chemist, the manufacturer, and the testing engineer will have to proceed simultaneously hand in hand, until the composition of the metal, its manufacture, and its capacities are thoroughly understood, and the uniformity of its production assured.

I also quite agree with the author, that the rate of speed with which strains are applied has much to do with the behavior of the specimen. Many hundreds of experiments have convinced me of this, and I would recommend that some basis be agreed upon as to the speed with which specimens of various sizes are to be tested.

As to the size of test pieces, I herewith submit a print, Plate XLIX, of the dimensions which we have adopted in our practice upon the Erie Railway. We have obtained the most satisfactory results from the long specimens.

The general attitude of engineers on the subject of bridge steel may be stated as one of expectancy. It is only of late years that it has been produced cheaply enough to warrant us in thinking of employing it at all; and while most of us believe it to be the material of the future, we are still inclined to put the burden of proof upon the steel makers, and to require them to furnish evidence of its adaptability and economy before we will agree to use it.

The bridge specification of the New York, Lake Erie & Western Railroad says in reference to steel:

"Sec. 14.—The following clauses are intended to apply to iron construction. Parties proposing to substitute steel for particular parts will be required to furnish evidence of its strength, elasticity, uniformity in production, and adaptability to the intended purpose."

The propriety of these requirements will be apparent, if we consider them in detail.

As to ultimate strength, we know that in tension steel ranges all the way from 50 000 to 300 000 pounds to the square inch which latter strength it has reached in the shape of fine wire. That in compression it begins to yield, or rather to be distorted, at from 40 000 to 90 000 pounds per square inch, while no one knows what its ultimate crushing point is for sections several inches in area, nor what weight will squeeze out flat even a single cubic inch of steel.

In fact, the great range in the strength of steel is one of the principal obstacles to its use. We do not know how much we can expect from it nor what strain it is safe to impose, and as prudence dictates that we shall assume the strength to be that of the weakest piece, we cannot, at present prices, generally use this metal in bridges with any marked economy until we know more about it.

There are a number of patent processes, more or less well known, of making steel by the addition of various metalloids, which are said to give uniform and satisfactory results. We know much less about them than is essential to their general employment, and hitherto the cost price has been so great as to prohibit the use of the metal in any but very large spans.

The author has judiciously insisted in his paper upon the importance of ductility in steel, and it may be pointed out in passing that much of the success of American iron bridge building has been due to our employment of soft, elastic metal, especially in all tension members. Although we take the utmost pains to secure accuracy of finish of the several parts—planing or turning all joints and bearings, boring all eye bars at the same temperature to fit the pins within one-fiftieth of an inch, and rejecting them if, when all the bars of one system are piled on top of each other, the pins do not pass through at both ends simultaneously without driving; although, in fact, we treat and finish our bridges as tools or machines, while in other countries they are treated as structures,

with the inaccuracies and defects inherent to that mode of construction, yet it may occur that when several members are placed side by side in a bridge, some of them will at first take a little (perhaps very little) more of the strains due to their position than others. Here the elasticity of the material comes into play. The bars which are first loaded yield a little and transfer part of their strains upon the others, until they are fairly equalized, and all substantially carry the same ratio of the load.

One difficulty is here likely to occur with steel. It is understood that its modulus of elasticity varies greatly, say from 25 000 000 to 35 000 000 of pounds, or a range of forty per cent., so that if bars with different moduli are placed side by side, it is not certain that the strains will be equalized among them as perfectly as now occurs with iron.

It is, however, in respect to the uniformity of production, the homogeneity of the material, that the greatest difficulties are to be surmounted. It seems to be understood that high carbon steel, made at the same works from the same materials, differs materially day by day in its strength and elasticity, and as we cannot well test a sample out of every bar, we cannot now be sure of just what strength we shall have in a bridge when we place the various bars side by side. It is true, that in boiler steel practical uniformity has been secured, and the product is homogeneous, but this contains only from $\frac{6}{100}$ to $\frac{10}{100}$ of one per cent. of carbon, and is substantially weldless iron of soft quality, while it costs five or six cents a pound, or enough to prohibit its use in bridges.

I have here tables of experiments (pages 284 and 285,) recently made upon boiler steel and iron for the New York, Lake Erie & Western Railroad, from which it appears that while the steel plates showed an ultimate strength, varying from 50 950 to 62 310 pounds per square inch, with a general average of about 52 000 pounds, and an elastic limit of 29 340 to 42 350 per square inch; the "Ulster iron" from which the stay bolts were made showed an ultimate strength of 48 000 to 52 000 pounds and the elastic limit of 28 000 to 33 000 ponnds per square inch, or from five per cent. to ten per cent. less than the steel.

I think in this matter we shall have to call upon the manufacturers to co-operate with the Society, by sending to the Committee on Tests numerous samples of their products of steel to be tested in small specimens by a uniform method, and to have the results published from time to time. Perhaps the interest thus awakened may induce Congress to

make a fresh appropriation for the testing of American iron and steel, so that we may have tests made upon full sized members, and thus acquire knowledge which we sorely need, before we can commit ourselves fully to the employment of this material.

The figures below give the dimensions of the specimens tested, and referred to in the following tables.

Fortunately, the use of steel will accord very well with the characteristics of American iron bridge practice, as it has been developed during the past fifteen years, creating a type differing materially from those of other countries, which has not inaptly been called the "skeleton type."

These characteristics differ from those of European bridges chiefly in the following particulars:

1st. The great depth which we give to our trusses in proportion to their length, endeavoring always to obtain the most economical depth for each span. Thus, the height of our bridges is generally from one-fifth to one-seventh of the span, while that of European bridges is from one-eighth to one-twelfth.

2d. The designing of bridges so that the direction taken by the strains and their distribution shall be absolutely certain, in order that the effect can be accurately calculated. There is no ambiguity or doubt as to the way strains are distributed in skeleton structures, while in plate

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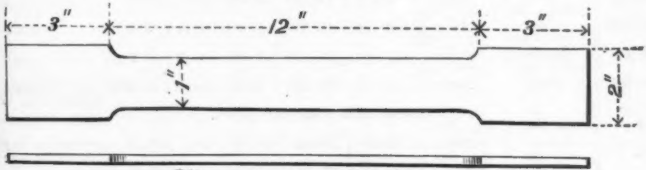
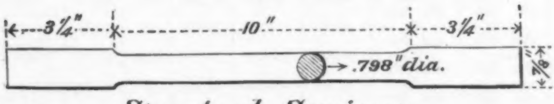
STEEL BOILER PLATE.



Standard Specimen

One-fifth full size.

STAY BOLT (ULSTER IRON).



Standard Specimen

One-fifth full size.

Tests of Tensile Strength of Material used in construction of Boilers for Consolidation Engines built by the Grant Locomotive Works, Paterson, N.J.

No. of Specimen and Mark.	Original Section.	Fractured Section.	Ultimate Strength per Square Inch. Original Section	Ultimate Strength per Square Inch. Fractured Section.	Elastic Limit per Square Inch.	Percentage of Elongation	Percentage of Red'n of Sect'n	REMARKS.
STEEL BOILER PLATE.								
	Inches.	Inches.	lbs.	lbs.	lbs.			
1 608	1.026 × .325 .333 □	.725 × .214 .155 □	50 982	109 571	29 990	21 5/16	53 13/32	
1 609	1.020 × .317 .323 □	.741 × .220 .163 □	52 576	104 281	33 401	22.3	49 3/8	
1 610	1.015 × .390 .396 □	.791 × .258 .204 □	57 597	111 721	35 998	16 1/8	48 3/8	Laminated.
1 611	1.010 × .390 .394 □	.750 × .256 .192 □	59 406	121 875	38 715	20 3/8	51 1/4	
1 612	1.020 × .498 .508 □	.926 × .422 .391 □	59 256	77 027	37 896	12 15/16	23 1/4	Laminated.
1 613	1.017 × .503 .511 □	.859 × .390 .335 □	60 795	92 833	37 142	12 13/32	34 1/2	Laminated.
1 614	1.025 × .508 .521 □	.730 × .344 .251 □	51 853	107 518	30 728	24 1/4	51 7/8	
1 615	1.021 × .450 .459 □	.690 × .270 .186 □	51 692	127 482	29 340	24 1/32	59 9/32	1608 to 1615 incl. Tested Sept., 1878.
1 677 T 1	1.015 × .335 .340 □	.869 × .218 .189 □	52 941	95 017	30 882	20 5/16	44.28	
1 678 T 3	1.009 × .325 .328 □	.820 × .188 .154 □	51 080	108 653	32 020	14 1/8	52.98	
1 679 B 5	1.004 × .325 .326 □	.758 × .205 .155 □	56 696	119 050	32 945	15 3/8	52 1/5	
1 680 B 6	1.005 × .332 .3336 □	.806 × .175 .141 □	50 950	120 524	30 720	20 1/8	57.72	
1 681 B 7	1.0 × .323 .323 □	.764 × .205 .1566 □	57 890	119 400	33 280	17 1/16	51 1/2	
1 682 L 2	1.0 × .325 .325 □	.767 × .208 .159	54 615	112 260	33 077	21 1/16	50.9	
1 683 L 4	.998 × .328 .327 □	.786 × .183 .144 □	53 460	121 660	32 840	20 2/3	56.06	
1 684 T 2	.996 × .338 .336 □	.822 × .284 .233 □	55 690	80 317	37 87C	9 1/8	30.65	
1 685 L 6	1.0 × .334 .334 □	.770 × .183 .141 □	52 395	124 190	33 680	20 11/16	57 81	
1 686 R 17	1.003 × .33 .33099 □	.76 × .175 .133 □	52 116	129 700	36 255	16 1/4	59.81	
1 687 R 8	1.002 × 323 .3236 □	.777 × .178 .138 □	56 388	131 950	41 600	16 7/16	57.26	
1 688 R 11	1.00 × .339 .339 □	.775 × .184 .1426	55 310	131 480	39 823	14 1/8	57.93	1677 to 1688 incl. Tested Jan., 1879.
ULSTER IRON FOR STAY BOLTS.								
1 644	dia. .798 0.5 □	dia. .580 0.2641 □	50 000	94 620	33 000	26.7	47.16	Indications of slag in these four (4) samples.
1 645	dia. .798 0.5 □	dia. .633 0.3147 □	49 000	77 850	32 000	21	37.06	Samples for 1st 10 engines, 1644 to
1 646	dia. .798 0.5 □	dia. .630 0.3117 □	49 000	78 560	28 500	22	37.65	1647 inclusive.
1 647	dia. .798 0.5 □	dia. .575 0.2596 □	49 500	95 310	31 000	26	48.06	Tested Nov., 1878.
1 660	dia. .798 0.5 □	dia. .619 .301 □	48 500	80 583	28 000	20.6	39.81	Indications of slag in these four (4)
1 661	dia. .798 0.5 □	dia. .588 .2715 □	48 500	89 302	30 500	26 3/8	45.69	samples.
1 662	dia .798 0.5 □	dia .625 .307 □	48 000	78 227	28 000	24	38.64	Samples for 2d 10 engines, 1660 to 1663 inclusive.
1 663	dia. .798 0.5 □	dia .612 .294 □	50 600	86 005	32 000	22 1/2	41.17	Tested Dec., 1878.
1 673	dia. .798 0.5 □	dia. .591 .274 □	48 400	88 215	27 500	24	45.13	Indications of slag in these four (4)
1 674	dia. .798 0.5 □	dia. .567 .252 □	52 000	102 970	30 500	24	49 1/2	samples.
1 675	dia. .798 0.5 □	dia. .582 0.266 □	49 000	92 095	29 500	24.5	46.79	Samples for 3d 10 engines, 1673 to 1676 inclusive.
1 676	dia. .798 0.5 □	dia. .588 0.271 □	48 000	89 302	29 000	21.6	45.69	Tested Feb., 1879.

Tested at Stevens Institute of Technology.

BOILER PLATE STEEL FOR N. Y. L. E. AND W. R. R.

No. of Specimen.	Material.	Original Section.	Fractured Section.	Ultimate strength per sq. inch.		Elastic Limit per sq. inch.	Percentage of		Original and Final Length.	REMARKS.
				Original Section.	Fractured Section.		Elongation.	Reduction of Section.		
		Inches	Inches.	lbs.	lbs.	lbs.			Inches.	
1742	Boiler plate steel......	1.011 × .314	.646 × .191						10	
		.317□	.123□	56 700	145 900	38 590	15.1	61.13	11.51	Marked, "Steel Boiler Plate."
1743	Boiler plate steel	1.013 × .319	.633 × .207						10	
		.323□	.131□	56 480	139 280	38 680	13.3	59.45	11.33	" " "
1744	Boiler plate steel	1.011 × .427	.675 × .26						10	
		.432□	.175□	57 330	141 030	35 380	17.7	59.35	11.77	" " "
1745	Boiler plate steel	1.006 × .428	.684 × .253						10	
		.430□	.173□	56 900	141 580	35 420	18.5	59.81	11.85	" " "
1746	Boiler plate steel	1.013 × .507	.72 × .331						10	
		.513□	.238□	60 360	130 080	36 510	15.6	55.6	11.56	" " "
1747	Boiler plate steel	1.011 × .508	.779 × .363						10	
		.514□	.283□	62 310	113 160	42 350	13.0	44.94	11.3	" " "
										Tested May 6th, 1879.

Tested at Stevens Institute of Technology.

girders and lattice bridges considerable material has to be added to allow for the possible imperfections of the riveting, and to prevent buckling.

3d. The concentration of the material into as few parts as possible, thus gaining the double advantage of reducing the number of connections (which merely transfer the strains from one member to another) to a minimum, and, consequently, saving both material and labor, and of generally making each member so large that it protects itself against weakening by oxydation and against accidental blows.

4th. As a consequence of the last two features, the general use of pins for connections. Even, however, when we use riveted connections, we preserve the great depth of truss, the skeleton design, and the concentration into few members by using large panels and making the compression members of *T* sections, thus building "trellis bridges" instead of the "lattice" bridges made of small, flat bars requiring vertical stiffness, such as have been so extensively used in Germany.

5th. The adoption of light, open floors of iron or wood, providing the necessary elasticity by wooden ties, and providing against danger from derailments by placing these ties about 6 or 8 inches apart in the clear, and surmounting them with a heavy guard beam, so that derailed wheels may run across the floor without either dropping through or "slewing" around, so as to allow the cars they carry to strike the sides of the bridge. These are equally safe, and far cheaper than the heavy ballasted floors, so common in European bridges, which are thought there of advantage in counterweighting the girder against partial loads, the necessity for this being obviated by our peculiar mode of construction.

These features have enabled us to perfect, in spite of some early failures, a general style of iron bridges, which is now admitted to be thoroughly efficient and safe when designed by competent builders, and which weighs from 10 to 25 per cent. less, for equal loads and strength, than the bridges built upon standard European types.

Now, in view of these features, what requirements and strains are we likely to adopt for steel bridges? We are all agreed upon the modes of calculating strains, and pretty well agreed as to specifications for iron bridges. Can we now agree upon a steel specification?

The author of the paper under discussion states in general terms that it would not be advisable to increase our customary working strains used for iron bridges more than 50 per cent., and I quite agree with him.

We now allow upon iron the following strains in tension :

	Pounds. per Square In.
On lateral bracing	15 000
" solid rolled beams, used as cross floor beams and stringers	10 000
" bottom chords and main diagonals	10 000
" counter rods and long verticals	8 000
" bottom flange of riveted cross-girders, net section	8 000
" bottom flange of riveted longitudinal plate girders *over* 20 feet long, net section	8 000
" bottom flange of riveted longitudinal plate girders *under* 20 feet long, net section	7 000
" floor beam hangers, and other similar members liable to sudden loading	6 000

This is upon metal which by the specification may range from 48 000 to 50 000 pounds per square inch for plates, angles, and channels, with an elongation of 15 per cent., from 50 000 to 52 000 pounds per square inch for small bars and rods, with an elongation of 18 per cent., and from 46 000 to 50 000 pounds per square inch, with an elongation of 15 per cent. for large flat bars.

We may say, therefore, that with the exception of floor beam hangers, and other similar parts exposed to great fatigue and impact with every passing load, we utilize in tension 20 per cent. of the ultimate strength of the iron ; or if we calculate it, as I believe to be more correct, upon the elastic limit of the iron, which is, generally, from 24 000 to 26 000 pounds to the square inch, that we utilize 40 per cent. of the strength beyond which it takes a permanent set.

If we should observe the same proportions for steel, we could place upon plates, angles, and channels (which the author proposes shall be of metal with an ultimate strength of 65 000 to 70 000 pounds per square inch), strains varying from 13 000 to 14 000 pounds per square inch ; upon small bars and rods, with an ultimate strength of 75 000 to 80 000 pounds per square inch, strains of 15 000 to 17 000 pounds per square inch ; and upon large, flat steel bars, with a strength of 70 000 to 80 000 pounds per square inch, strains of 14 000 to 16 000 pounds per square inch.

Or if we base our strains upon the elastic limit, which the author proposes shall be 35 000 to 40 000 pounds, we could, by utilizing 40 per cent. of this, strain the steel from 14 000 to 16 000 pounds per square inch.

This agrees pretty closely with the result arrived at by adding 50 per cent. to the strain allowed on iron in tension, as previously stated, giving, (if we omit the floor beam hangers, the bottom flange of riveted plate girders under 20 feet long, and the lateral bracing,) strains allowable for steel, of 12 000 to 15 000 pounds per square inch.

An incidental advantage, however, is likely to result in some parts of bridges, from the use of steel in tension, by the opp tunity which the greater strains allowed will afford for reducing the number of parallel parts of members dividing the duty between them, and thus making our bridges more compact, as well as insuring a more accurate distribution of the loads among the fewer pieces which are expected to pull together.

With respect to compression members, we are woefully ignorant of their real strength in full-sized members, mainly because until quite recently there has been no reliable machine strong enough in this country to cripple top chord pieces and large struts. The Government machine, which is just done, could give us almost invaluable information on this subject, but unfortunately the want of an appropriation has locked it up beyond reach.

There is reason to suspect that in our ignorance of the real crippling or distortion point of large compression members, and of the best form to be given to them, we are now underloading them, and wasting considerable quantities of material.

4.—In beams and girders compression is limited, as follows:

	Pounds per Square In.
In rolled beams, used in cross floor beams and stringers...........	10 000
" riveted plate girders, used as cross floor beams, gross section...	6 000
" " longitudinal plate girders *over* 20 feet long, gross section.	6 000
" " " " " *under* 20 ft. long, gross section.	5 000

In top chords, posts, and struts, the strains are calculated by a modification of Rankine's formula, as follows:

$$P = \frac{8\,000}{1 + \dfrac{L^2}{40\,000\,R^2}}$$ for square end compression members.

$$P = \frac{8\,000}{1 + \dfrac{L^2}{30\,000\,R^2}}$$ for compression members with one pin and one square end.

$$P = \frac{8\,000}{1 + \dfrac{L^2}{20\,000\,R^2}}$$ for compression members with pin bearings.

P = the allowed compression per square inch of cross section.
L = the length of compression member, in inches.
R = the least radius of gyration of the section, in inches.

The lateral struts are proportioned by the above formulæ, to resist the resultant due to an assumed initial strain of 10 000 pounds per square inch upon all the rods attaching to them, produced by adjusting the bridge.

This gives from 8 000 to 4 000 pounds per square inch of allowed compression upon square ended members, as usually proportioned.

Now, it must be remembered that this is based upon the *crippling point* of small members only, such as we can test in ordinary machines, and that not only are we not sure that large members will behave quite in the same way (in fact we know that both in stone and iron, cubes of, say 4 inches, are more than 16 times as strong in compression, than cubes of 1 inch); but to be on the side of safety, we utilize but 20 per cent. of the assumed crippling strength, which really corresponds to the elastic limit in tension, of which we utilize 40 per cent.

When we come to use steel it may be wise to consider whether it wil not be safe to increase materially the compressive strains which we shall allow.

If the formula quoted above is correct for American iron it gives a crippling point varying from 40 000 pounds for 1 diameter to 23 000 pounds per square inch for 70 diameters, and we may, perhaps, find it safe to utilize in steel, something more nearly approximating 40 per cent. of the strains at which distortion begins,

The final test by which the use of steel shall be determined must, of course, be its economy over iron. If we assume, in a general way, that it will cost no more for manipulation in the shop, for transportation and for erection than iron, we may ascertain approximately the price at which a reliable uniform product must be sold in order to enable it to compete.

The base price of bridge iron is now about 2½ cents per pound, and if it is determined that it can be strained on an average 50 per cent. more than iron, it must be sold at 3¾ cents a pound to make its economy equal. It is for the manufacturers to say whether they can give us a reliable, ductile and safe steel at this price.

We cannot too strongly insist, however, upon the importance of homogeneity and uniformity in production. We must have a material

upon which we can rely as generally as we now do on iron, and we must understand its peculiarities and behaviour under manipulation thoroughly.

To obtain this knowledge there seems to be no way open but careful experiment, first in the testing room, and then in actual construction. It may possibly be that after all, steel shall not prove more economical than iron for bridge construction, but the chances of its becoming so will certainly warrant the trial.

For long spans, say those over 350 feet, where the dead weight of the bridge is greater than the moving load, it is probable that steel is already cheaper than iron for the main members, because it saves in its own weight, to the extent of the greater strains, which it will be found safe to impose upon it.

Such great spans, however, are rare, and it should be the endeavor of those who may be interested in bringing steel into use, so to develop its capacities, establish its safety, ensure its uniformity of production and cheapen its cost, as to enable us to make new specifications which will insure its substitution for iron in spans of, say 100 to 200 feet, such as are in daily demand.

D. Torrey.—What I have to say relates more to metallurgy than to engineering, and while really in the same line of study and research as the greater part of the papers before us, is yet so pertinent to the problem of steel bridges as to be acceptable, I trust, to you.

In giving my opinions, it should be clearly understood that they are offered as conclusions based upon experience not specially related to bridge building, as may be the case with others who share in the discussion. In a word, we are looking forward, not summing up. It is safe to assume that the one great difficulty preventing the general adoption of steel as a bridge building material, is the wide-spread belief that the metal is, to use a common expression, treacherous. The frequency with which pieces of steel, purchased with a guarantee of high tensile strength and superior quality, have unexpectedly broken, either while being manufactured or while in service, and this in positions that iron has filled much better, has carried conviction to the minds of many engineers that its use in bridge construction involved too much risk. This conviction has been strengthened by the absence generally of any fair explanation of the cause of failure, or the honest confession given by saying, "I don't understand it." Laboratory tests, which show how very much superior steel

is to iron, are of no value in seeking a solution of the problem; and the greater the degree of confidence with which a man tries steel as a substitute for iron, the more emphatically does he reject it when it fails to serve his purposes so inexplicably as it sometimes does.

What is most wanted is some way to make steel safe for structural purposes, without also making it too costly. Why is it unsafe? Probably because it is not handled with sufficient care in the process of manufacture, and during subsequent manipulation and transportation; and what I have to suggest is the preparation of steel in a condition to be much less sensitive than it usually is to these adverse influences.

Many persons do not know how difficult it is to secure uniformity in quality with any considerable number of pieces of steel. Blooms from the same ingot, when reheated, vary in carbon according to their positions in the heating furnace, as also the length of time they lie in it; and they vary in physical characteristics according to the manipulation they receive—variations depending upon temperature when taken to the rolls, speed of rolling and conditions of cooling, etc.

The surface defects of a piece of steel, such as hammer marks, cracks, etc., even when invisible to the eye, may cause it to break if the piece is subject to vibratory strains, when a like piece of iron would not fail. The milder the steel the less the cohesion between its particles, and the less tensile strength, and while this ready adjustment of its molecules under stress fits it for bridge and other structures, its low tensile strength gives it little advantage over iron. High tensile strength steel will probably resist the appearance of incipient fracture better than low tensile strength steel, but when the incipient fracture is present the high steel will be much the weaker. High steel, when mechanically perfect, that is to say, without surface cracks, however fine, and without internal flaws or external hammer marks, will, from its greater molecular cohesion, resist vibratory strains better than low steel.

This has been illustrated in the experience of Mr. William Metcalf with the piston rod of a steam hammer, who reports that while wrought iron endured three months, mild steel six months, high steel, 60 carbon, endured two years. These pistons were too large to crush and were not subject to flexure; they were always broken.

By ordinary processes of manufacture mild steel has porous cavities in the ingots. Some authorities say it is never without them, and as it will not bear heating to a temperature high enough to enable us to weld

the walls of these cavities in the subsequent process of rolling or forging, when once present they persistly remain, and may constitute incipient fractures.

If we should take a piece of steel free from blow holes or cavities, form it into a desired article for bridge or other structures, anneal it, and then dress its entire surface in a planer, giving it a polished surface, we should have a nearly perfect bridge building material; but as this would cost too much, it is desirable to see how we can most nearly attain the same result within practicable limits of cost and manipulative skill.

It is proposed to confer upon steel intended for structural uses the desirable qualities possessed by iron, by covering the steel with a shield of iron, which iron will protect the steel from surface injuries during the processes of manufacture and application. In a word, to make iron-clad steel for bridge building. By doing this we secure the effectual effacement of porosity and its train of evils; we secure a surface of metal in a condition that cracks, crevices, hammer marks, &c., will not become foci of vibration followed by fractures to a degree greater than with articles wholly made of iron. We gain incidentally a condition fitting our material for exposure in the furnace, by which we can obtain greater uniformity of carbon in the steel, and finally our metal is less sensitive to the evils of rough treatment through all the stages of manufacture and application. That is iron-clad steel, higher in carbon than thought safe for structural uses as naked steel, can be cut, punched and bent in a manner not admissable with the naked steel.

Besides commanding these advantages, a most important matter is the fact that iron-clad steel should be furnished cheaper than ordinary steel.

Theodore Cooper.—In closing the discussion upon this subject, I regret that certain points advanced by me, which I believe have two sides, were not questioned.

Two of the members have drawn attention to the use of high steel for steam hammer rods and similar uses as being of an opposite character of material from that I have claimed as proper for bridge purposes.

The writer has studied the experiments of Mr. Wm. Metcalf with much interest, and thinks he has made out presumptive evidence in favor of high steel for such purposes as he recommends. But he does not see the connection between the uses of high steel for resisting vibrations,

and low steel which must resist the stretching and compression due to the mechanical operations of the shop.

That a high steel would best resist the vibratory action induced by sudden changes of strains might almost appear as certain if we had not had it confirmed by experiments. The more elastic the material and the greater its range of elasticity—the less chance for its being overstrained and thus injured.

But no experiments have been made to show that high steel in irregular forms would be able to resist such repeated vibrations. On the contrary, all our experience goes to show that any sudden change of section in high steel is perfectly ruinous.

It will not stand the rough mechanical manipulations of punching, bending, straightening, &c, which bridge material has to go through. Only the mild, soft steel will give any favorable result under such treatment.

Mr. Macdonald says, why cannot we use 20 000 pounds tension upon high steel instead of the figures given in the paper? Because to use such a high unit of strain you must have a material which will not safely bear the treatment which *iron* bars have to pass through, and it is not justified by our present knowledge of the material, unless we use short pieces, and give them much more careful treatment than we do bars of iron which have a high ductility. The writer has not limited the strains in any other manner than by insisting upon a ductile material. If this can be obtained, the higher the tensile strength the better.

Mr. Torrey has put forward an iron-clad steel. This can only enter the competition as an unknown material, and must prove its capabilities before it can be considered as either iron or steel, of which we do know something. Whether two materials of different strength and ductility can be combined into a practically useful material is very doubtful, in the writer's opinion.

Mr. Clarke has presented us the results upon the use of steel for bridges in Holland. If he could also have given us the specified terms in regard to the quality of the material it would have been very instructive. It would appear as if the steel was the very kind considered in the preceding paper as unfit for bridge purposes. The plates used in these Dutch bridges were required to stand, *without* breaking or tearing, a strain of 82 000 pounds per square inch for a length of 15 minutes, which would require a material of *over* 90 000 pounds per square inch tensile

strength. Now, at the time these bridges were built, this would indicate a material that would have but little ductility.

No one would be more pleased than the writer could *experiment* show that we could venture safely into the use of high steel, but without convincing proof deduced from extended experiments it would, in his mind, appear very hazardous to attempt to throw away the factor of the ductility of iron and steel.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

CLXXXVII.

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THE CONSTRUCTION OF

THE ATCHISON, TOPEKA AND SANTA FÉ RAILROAD

OVER THE RATON MOUNTAINS,

AND THE PERFORMANCE OF LOCOMOTIVES ON ITS STEEP GRADES.

By JAMES D. BURR, C. E., Member of the Society.

PRESENTED AT THE ELEVENTH ANNUAL CONVENTION, JUNE 17TH, 1879.

The railroad over the Raton Mountains, of standard 4′ 8½″ gauge, is a branch of the Atchison, Topeka and Santa Fé Railroad, under the names of Pueblo and Arkansas Valley, in Colorado; and New Mexico and Southern Pacific, in the Territory of New Mexico.

The Raton Mountains, a spur of the Sangre de Christo, or Snowy Range, extend nearly due east and west; while the Snowy Range, a portion of the Rocky Mountain system, extends almost due north and south, parallel to the direction of the road. The Ratons culminate in Raton (or Fisher's) Peak, 9 800 feet above the level of the sea, while the summit of Raton Pass is reached by the grade line at an elevation of 7 720 feet above the same plane.

A cross-section of the country from La Junta, Col., in the Arkansas Valley, to Las Vegas, New Mexico, over the line of the constructed road, is shown on Plate L. A map and profile of the Mountain Division, Trinidad to Willow Springs, is shown on Plate LI.

Plate LII, is a map and profile of the temporary track, or "Switchback."

Table 1 gives the distances, grades and curvatures from La Junta to Las Vegas.

Table 2 gives a statement of the progress of Raton Tunnel, kindly furnished by R. L. Engle, C. E., Engineer in charge of the construction of the Mountain Division, including Tunnel and "Switchback."

Plate LIII shows the Consolidation Engine, "Uncle Dick."

La Junta, Colorado, 555 miles west of Atchison, Kansas, at an elevation of 4 041 feet above tidewater, is the point where the Trinidad Branch of the Pueblo and Arkansas Valley Railroad diverges from the main line. This Branch extends to the boundary line of the Territory of New Mexico (97 miles from La Junta), at which point the New Mexico and Southern Pacific Railroad begins. This latter road is now nearly completed as far as Las Vegas, N. M., a distance of 118 miles from the boundary line.

From La Junta, Col., to Trinidad, a distance of 81 miles, maximum ascending grades of 60 feet per mile have been used, with adverse grades of 30 feet per mile. Curves of 1 146 feet radius are used, with a compensation or reduction of grades on curves at the rate of .05 foot each 100 feet, for each degree of curvature.

To preserve uniformity throughout, the following rules were adopted:

1st. Generally, the point from which to commence elevating the outer rail on curves, shall be on the tangent, 150 feet from point of curve, and shall attain its proper elevation at the point of curve.

2d. In cases of compound curves, or curves in the same direction where there is less than 300 feet tangent between curves, the same rule will apply,

3d. In cases of curves in reverse direction with only 100 feet tangent between them, the elevation shall begin at the point of tangent to the curve on the inner rail and attain its proper elevation at the point of curve on the outer rail, of the succeeding curve, and so, in an opposite sense, for the other rail.

There are thus on curves of this class, two planes inclined in opposite directions relative to the true grade line.

TABLE 1.

DISTANCES, GRADES AND CURVATURE FROM LA JUNTA, COL., TO LAS VEGAS, N. M.

	Distance in miles.	Total Rise.	Total Fall.	Maximum Gradient in feet per mile.	Maximum Curvature degrees per 100 ft.	Total Curvature	Average Gradient.	Elevation Outer Rail per degree.	Compensation for Curvature in ft. per degree.	REMARKS.
								Inches.		
La Junta to Trinidad.........	81.5	....	....	−30. +60.	5°	....	....	0.5	0.05	It is difficult to estimate the rise and fall of an engine with train, owing to the difference in length of train on the different stems.
Trinidad to Morley...........	10.1	800.	1.5	105.6	10°	865.1°	80.	0.5	0.05	
Morley to State Line..........	5.4	818.	....	184.8	10°	945.°	151.4	0.5	0.05	
State Line to Willow Springs..	7.67	43.	990.	+184.8 −175.3	10°	1047.°	132.	0.5	0.05	
Willow Springs to Las Vegas..	110.25	....	....	70.	6°	....	....	0.5	0.05	
Switch Back..................	2.75	280.5	280.9	316.8	16°	1081.°13'	221.8 189.0	0.25	....	

Plate L will show that there are but three short planes of descending gradients, of any considerable moment, between La Junta and Trinidad, going south, and as it was expected that there would be a large traffic in coal from Trinidad to supply the territory for 300 miles to the northeast, it was desirable to make the ascending grades, going north, as light as possible, consistent with economy in construction. In order to locate a line on 30 feet ascending grades to the north, from La Junta to Trinidad, —compared with the location of the Kansas Pacific Surveys made in 1871–72 on 70 feet maximum gradients *each* way,—a loss of 3 000 feet in distance was sustained.

From Trinidad the line follows the Valley of the Purgatoire—Picket-wire, in frontier language—two miles to the mouth of Raton Creek. Thence, turning sharply to the south, with both maximum curvature and maximum gradients, it ascends the northern slope of the mountain proper by Raton Cañon, on two planes.

The first plane from the mouth of Raton Creek to Morley, eight miles, is nearly a uniform ascent of 105.6 feet per mile. At Morley, with an elevation of 6 727 feet above tide-water, a water-tank, turn-table and side tracks have been established. The yard is on a gradient of 1.66 feet per 100, or 87.65 feet per mile. At the south end of this yard the second incline plane commences, having 3.5 per cent. maximum grades. Between Morley and the summit there are three miles of maximum, supported grade. The average ascent is 151.4 feet per mile.

The summit is passed by a tunnel which, when completed, will be 2 011 feet long, located on 1.9 per cent. ascending gradient, going south. The south portal is 7 584 feet above the sea, and the top of the mountain proper has an elevation of 7 767 feet. Thus, in 306 stations of 100 feet each, there is an ascent of 861 feet, or a uniform ascending gradient of 2.8 feet per hundred. At the south portal of the tunnel the line commences to descend the southern slope of the mountain, on 3.32 per cent. maximum gradients, to Willow Springs, which has an elevation of 6 595 feet above tide, having made a descent of 990 feet in 38 400 feet, or a uniform gradient of 2.58 per cent.

On the mountain division, from Trinidad to Willow Springs, maximum curves of 573.7 feet radius are freely used. Maximum grades are compensated for curvature at a rate of .05 feet per degree of curvature, each 100 feet. The outer rail has been elevated at the rate of ½ inch per degree of curvature.

From Willow Springs to Las Vegas, "The Meadows," 110 miles beyond, the line has been located on 70 feet maximum gradients with 6° curves from maximum curvature, on which the work of construction has so far advanced that Las Vegas will be reached early in July.

The surveys from La Junta to Willow Springs were begun on the northern slope of the mountains early in March, 1878, by A. A. Robinson, Chief Engineer, with George B. Lake (now Sup. Western Division) First Assistant Engineer, who made the location on the north side, while the location on the south side was made by R. L. Engle, Division Engineer in charge of tunnel. A company force was at once organized, and grading commenced.

The excavation at each end of the tunnel being very deep, 56 feet at the north portal, and 50 feet at the south end, mostly in solid rock, a shaft near the south portal was begun on June 1st. The shaft reached the roof of the tunnel section, July 9th.

On August 21st, the north approach had progressed so far that the heading was commenced at that end, but up to August 31st, only 73 feet of heading and 19 feet of full section had been driven, 14 feet of the latter being that allowed for the section of the shaft.

At this time the track had reached a point 65 miles south of La Junta, and it became evident that the completion of the tunnel must be hastened or a temporary track built over the mountains, otherwise track would be detained at the tunnel several months. It was, therefore, decided at the first meeting of any Board of Railroad Directors held in New Mexico, August 31st, 1878, to build a temporary track over the mountain, to be used until the tunnel could be completed. A location for a "switchback" was made immediately, and the grading for it, begun on the 16th of September, was completed in the latter part of November.

The maximum grade on tangent on the mountain top line is 6 feet per hundred; curves of 359.3 feet, least radius. Gradients are compensated for curvatures, on the switchback, as follows:

On curves of 1 146 feet radius and over, 0.05 foot per degree of curvature.

On curves of less than 1 146 feet radius to curves of 573.7 feet radius, 0.075 foot per degree of curvature.

On curves with shorter radii than 573.7 feet, 0.086 foot per degree.

The descent of the south slope is on a maximum of 4.9 per cent. on

straight portions of the track, compensated for curvature at the same rate as on the north side.

It will be noticed by reference to Plate LII that the ends of each stem (of which there are six, counting the two constituting the ends of the main line), are approached by ascending grades. This practice has proved a very desirable precaution; for, during the progress of construction a portion of a train became unmanageable on account of the wheels sliding down the frosty rails, and would have gone beyond the stem, and down an embankment many feet high, but for the resistance of the adverse grade. On May 12th an engine did go over the embankment at the end of Stem No. 2, on account of the brakes on the train giving way—although the engine was provided with air brakes on drivers—and there would have been a very serious accident, but for the resistance of the adverse grade. Fortunately the damage was slight, and only to the engine.

Before the close of December, 1878, the track had been laid over the mountain, and the work of transporting material for the construction of 118 miles of the New Mexico & Southern Pacific Railroad began. Besides this material, since the road has been opened for traffic, a period of three months, merchandise freight to the value of from $150,000 to $175,000 has been transported to and from the town of Otero, 12 miles beyond the summit. The average tonnage transported over the Switchback daily, since it was completed, may be stated, including dead weight, as, Tonnage South, 420 tons per day; Tonnage North, 200 tons per day.

At first the Switchback was operated by the ordinary eight-wheeled American engines with 17″ × 24″ cylinders, 60″ driving wheels, weighing about thirty-five tons, from the Baldwin shops, and with a 16″ × 24″ engine, same class as the first from the Hinkley shops, with 64″ drivers. The working of these light engines may be summarized as follows:

La Junta to Trinidad,	60. ′	grades,	14	to	18	loads	of	43,000	pounds.
Trinidad " Morley,	105.6′	"	8	"	11	"	"	"	"
Morley " Tunnel,	184.8′	"	3	"	5	"	"	"	"
" Switchback,"	316.8′	"	1	"	2	"	"	"	"

The smaller number of cars being the number for the 16″×24″ cylinder engine; and the larger number of cars, the maximum capacity for the 17″×24″ cylinder engines.

The best day's performance of any of these light engines on the mountain division was that of Locomotive No. 106, from the Hinkley shops, with 16″×24″ cylinders, 64″ drivers. This engine left Trinidad, empty, at 7 A. M., for the Summit, 15 miles distant, and, during the day, took twenty-five loaded cars from the North to the South Siding, 2¾ miles, and brought back as many empty cars, and arrived again at Trinidad at 7 P, M. This was accomplished by taking one load from North Siding and leaving it at the summit; thence returning to North Siding for a second loaded car, and again, for its third load; then, from the summit, taking the three loaded cars to the South Siding. This performance was at the time considered remarkable.

The engine was provided with Westinghouse Air Brakes on driving and tank wheels, as are all engines which work on this mountain line. Engines which were fitted with the Automatic Brake have had the arrangement changed into the common Westinghouse Air Brake; it having been found that this last gives the best results in regulating the amount of force applied to the air brakes, and the air gauge of the air brake always indicating the pressure in the reservoir before the brakes are applied, as well as during the operation. On the other hand, with the automatic device it is impossible to tell the pressure until, and, during the application of the brakes, unless the pressure is kept up to the blowing off point. A very essential thing to know, in working the Mountain Top line, is that the brakes and brake power are in good condition and under perfect control.

The performance of two coupled engines on the Mountain Division has not been double that of engine 106, whose remarkable day's work was made at great risk; but, from 7 A. M. to 7 P. M., 34 loaded cars have been taken over from the North to the South Siding, besides 10 loaded cars hauled from Trinidad to the Tunnel, the engines returning to Trinidad within the time mentioned.

The advent of the "Uncle Dick," a "Consolidation" eight-wheeled-connected engine from the Baldwin shops, revolutionized transportation on the "Mountain Top" line.

The principal dimensions of this locomotive are :

Cylinders	20″×26″	Number of Tubes	213
Driving wheels	42″	Diameter "	2″
Total wheel base	22′—10″	Length "	10′—11¾″
Rigid "	9 feet.	Heating surface, fire box,	153□′
Diameter boiler, inside	57″	" " flues	1223.84□′
Length fire box	119″	Total heating surface	1376.84□′
Width "	33⅜″		

Capacity of tank on boiler .... 1,200 gallons.
Estimated weight, including weight of saddle tank in working order .... 115,000 pounds.
Estimated weight on drivers .... 100,000 "

In this connection, it may be proper to say that "Uncle Dick" Wootten, the old pioneer after whom the locomotive is named, settled in the Raton Cañon in 1847 ; that he was with Fremont, Kearney, and Kit Carson ; and that he saw Fisher when he made his observations from the peak that bears his name.

The performance of this engine on the mountain division is as follows :

On the 2 per cent. incline, Trinidad to Morley—482½ tons hauled 8 miles per hour.

On the 3.5 per cent incline, Morley to Tunnel—258½ tons hauled 8 miles per hour.

On the "Switchback"—194 tons hauled 6 miles per hour, including time lost in opening and closing six switches. The foregoing weights do not include the weight of the engine.

On the 2 per cent. gradients the resistance is :

$$\begin{array}{llcl} \text{Gravity}\ldots\ldots & \dfrac{2\times 2000}{100} & = & 40 \text{ pounds per ton.} \\ \text{Wheel friction} & & = & 6 \quad " \quad " \\ \text{Wind pressure} & \text{(say)} & = & 1.8 \quad " \quad " \\ & \text{Total resistance}\ldots & & 47.8 \quad " \quad " \end{array}$$

Traction $= 482.5 +$ weight of engine say 60 tons $= 542.5$ tons.

Then, traction $= 542.5 \times 47.8 = 25{,}931$ pounds.

Weight of engine on drivers $= 100{,}000$ pounds.

$$\text{Adhesion} = \frac{25{,}931}{100{,}000} = \frac{1}{3.86},$$

or somewhat greater than ¼ the insistent weight.

On the 6 per cent. gradient:

$$\text{Gravity} \ldots\ldots \frac{6 \times 2{,}000}{100} = 120 \text{ pounds per ton.}$$

Wheel friction	= 6	"	"
Wind pressure	= 1.8	"	"
Total resistance....	127.8	"	"

Traction $= 254 \times 127.8 = 32{,}461$ pounds.

$$\text{Adhesion} = \frac{32{,}461}{100{,}000} = \frac{1}{3.08},$$

or, slightly less than ⅓ the insistent weight.

The difference between the traction on the 2 per cent. and on the 6 per cent. gradients is explained by the fact that the locomotive has not been tested to her full capacity on the 2 per cent. gradients. The loads mentioned above were started from a perfect stand-still, and without taking the slack of the train, and without slipping drivers.

The best day's work on the Mountain Top Line with the locomotive Uncle Dick has been as follows: Left Trinidad for Morley at 7 A. M., with 15 loaded cars, besides tanks of coal and water; drew 10 loaded cars from Morley to Tunnel, and during the day took 46 loaded cars from the North to the South Siding, and brought back as many empties; reached Trinidad at 7 P. M. Time, 12 hours, of which 2½ hours were lost in waiting for trains and for meals.

The ordinary round trip, 5½ miles, requires 50 minutes time. The ordinary train consists of 7 loaded cars of 43,000 pounds each, tank of coal 44,000 pounds, and engine say 120,000 pounds. Eight loaded cars can be taken over at one time quite readily; and, at one time, nine loaded cars were taken at one trip, so that, during the day of ten hours, 6,020,000 pounds could, very readily, be moved over the mountain with one engine. It will be noticed that the capacity of engines of this class is more than double that of two American engines of 16″ × 24″ cylinders; while the quantity of coal consumed is but little more than that consumed by a single light engine.

As to the matter of controlling trains on these steep inclines it is a question of brake power and adhesion to the rail. In frosty weather, backing down an incline, the ordinary eight-wheeled engine, not having a sand pipe in the rear of its drivers, will lose its "bite" or adhesion to the rails, and slide down hill quite as readily as if placed on runners on

good sleighing. In such cases brakes are of small account. All engines should be provided with sand pipes placed so as to deliver sand on the rail, when going in either direction.

Under fair conditions of rail, one single hand-brake, in good order, to each car, together with the driver and tank brake, with three brakemen to a train of eight cars, is sufficient for safety, unless the train should acquire a speed of 18 to 20 miles per hour, in which event all the wheels in the train might be skidded far enough to lead to disaster.

With the Uncle Dick this danger is reduced to a minimum, for, as sand can be delivered along the rail in front of all the wheels, and two sets of air brakes may act on all drivers simultaneously with sufficient force to slide all the wheels, the maximum of adhesion is obtained.

The Uncle Dick is provided with water pipes for delivering a small jet of water on the flanges of each flanged driving wheel. In backing over the sharp curves of the "Mountain Top" line this jet of water is very beneficial in preserving the flanges of the drivers and also the head of the rails.

The "Switchback" is laid with 56 pounds iron rails, common splices, pine ties on straight line, oak ties at the joints of rails, and on all curves having less than 1,146 feet radii. A cast iron bracket fits up against the outer side of each rail on curves with 716 feet, and less, radii, while a guard rail of iron is placed on the inner side of the inside rail. These precautions hold the track in gauge.

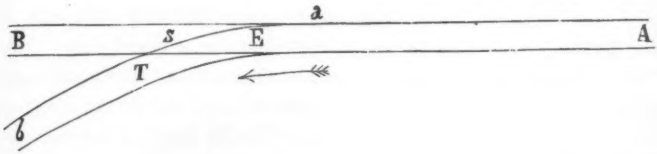
The gauge of rails is 4′—8¾″ on straight portions of the track to 4′—9¾″ on the 16 degree curves.

The outer rail on curves was elevated ¼″ per degree of curvature per hundred feet. The track was operated with this elevation, for some time, when, it being noticed that the inner rail was becoming much kinked, an elevation of ⅛″ per degree was given. This proved to be an error, as, with this elevation, the Uncle Dick seemed inclined to leave the rail in the direction of a tangent to a curve, even at slow rates of speed; therefore, the original elevation of ¼″ per degree, on curves, was restored. This has given the best results.

During the first two weeks' work of the Uncle Dick its performance was anything but satisfactory owing to the frequency with which it left the rails at frogs and switches. This was caused, at the frogs by insufficient width of gauge and the inability to elevate the outer rail; at the switches partly by the insufficient weight of rails, partly by the

small amount of play to the switch-rod connections, and very much by the great length and leverage between the trailing axle and the centre of the truck wheels under the tank.

From the plate of the locomotive Uncle Dick it will be seen that the original intention was to operate it as a tank engine. It was found, upon trial, that this could not be done with economy, therefore the ordinary tank was attached, and air brakes supplied in the usual manner.



If we suppose the figure to represent a portion of track near a switch with a descending grade from A to B, with the tank at T, engine at E, and the loaded train from A to a, moving in the direction of the arrow, and let the air brakes be suddenly applied to the tank wheels, the resultant of the two forces would throw the trailing wheel over against the slide rail S with sufficient force to cause a "lip" on the lead rail, and the engine would climb the lead rail and go over on to the main track.

This was obviated, in a measure, by taking off the coal bunker, reducing the length between engine and tank, and putting an additional set of air brakes on the drivers. At first there was but one pair of air brakes on the engine—those between the rear and next pair of drivers. These were used as little as possible, tank brakes being used instead, in order to preserve, as nearly as possible, the same diameter for all the wheels until the second pair of brakes could be supplied.

Since the foregoing defects have been remedied as much as possible, there has been no difficulty in working the Mountain Top Line, with the heavy engine. Indeed, it is believed that the Consolidation engine travels the 16 degree curves with as much ease as an ordinary American engine, and causes less wear of track and permanent way.

DISCUSSION

ON THE PAPER "THE CONSTRUCTION OF THE ATCHISON, TOPEKA AND SANTA FÊ RAILROAD OVER THE RATON MOUNTAINS," AT THE MEETING OF THE SOCIETY, OCTOBER 1ST, 1879,

By J. FOSTER FLAGG and EDWARD P. NORTH.

J. FOSTER FLAGG.—I have been looking forward with considerable interest to the appearance of the paper by Mr. Burr, but must confess to some disappointment in not finding therein more details of location and construction. I may, however, have overrated the difficulties of the former, which does not need, therefore, any especial description. The work of locating the developments of the track and the switch-backs of the Oroya Railroad, on the precipitous sides of the Rimac Valley in Peru, was one of exceeding difficulty,—which, by the by, I was in hopes we should have had a full description of long since from Mr. V. G. Bogue, Member of the Society, the engineer in charge of a division in the most difficult part of the route, he having intimated to me an intention of preparing a paper to this end several years ago,—and I may have been erroneously led to suppose there were similar difficulties here.

The details of grade and curvature are, however, quite valuable in connection with the performances of the consolidation engine, as a guide in estimating the possible capacity of roads in mountainous localities, where excessive grades and curvature have to be resorted to, whether as a temporary expedient or for permanent location. If the actual expenditure of fuel, etc., were given, we could also estimate closely the cost of transportation per ton mile.

On the Iquique and La Noria Railroad, in Southern Peru, and doubtless in other localities, the same expedient of laying the stem of the switch-back on a sharp up grade is resorted to, and its importance is very great in preventing or diminishing the number of accidents from runaway trains. This road has maximum grades of 4 per cent. only, but the great length of continuous steep descent makes the control of trains fully as difficult, probably, in the winter season, as upon the line over the Raton Mountain. From the station of Santa Rosa to the sea at Iquique, a distance of 17.2 miles, the track falls nearly 2 900 feet, an average of about $3\frac{2}{10}$ per cent. for the entire distance. Proceeding downwards from Santa Rosa, at the end of 7.7 miles, at Molle, the track,

being then about 1 600 feet above the sea, strikes the face of a steep cliff falling nearly to the ocean beach, and follows the face of the cliff for 7.5 miles to the only reversing station,—the remaining two miles having slight grades. There is thus a constantly descending grade for 15.2 miles, the average being at least $3\frac{4}{10}$ per cent. for this distance, with a maximum of 4 per cent., and very little less than 3 per cent. Although there is neither rain nor frost in this locality, there is in winter a very damp fog, which seems to impart a peculiar greasiness to the rails, making it very difficult to hold the trains, especially if they have once acquired any speed: and a train having fairly entered upon the *cuesta*, or face of the cliff, with a high velocity, the long distance to the reversing station, with numerous curves of 12° to 16° curvature on the way, renders its destruction almost certain.

After several severe accidents had happened on the *cuesta*, a switch was built at its head, on the pampa at Molle, to catch any runaway trains from above. This, like the switch-back, was built with a heavy ascending grade, with a self-acting split-rail switch, set to throw all descending trains upon the turn-out, and requiring the switch-tender, after throwing over the points *to hold them there* during the passage of a train in order that it might continue on its course down the main line. Notwithstanding this safeguard, a short time subsequent to its construction, by some stupid misunderstanding of signals, or lack of proper system in signaling, a train, of which the driver had lost control, came thundering by whilst the switch tender held open the path leading to its destruction. The result was an accident, which, with their insufficient appliances, blocked the road for five days, and a loss of life that would be appalling if its numbers could be stated; but the native railroad authorities managed to get the dead bodies of the poor friendless *peones* disposed of in some way, so that the amount of slaughter was never known.

At Sta. Rosa Station is a piece of level track, nearly half a mile in length, and above and beyond is the most difficult stretch of track on the road. Here, within a distance of 0.7 miles, are eight consecutive reverse curves, with no perceptible tangents between, with radii of only 330 to 570 feet—mostly under 400 feet—and with a total curvature of 535 degrees; it is all on a heavy ascending grade, a portion in the midst of the curves being 4 per cent.

In 1872 to 1874, when I was acquainted with the road, the locomotives were mainly Fairleigh's double enders, and it is interesting to compare

the performance of these much vaunted engines with that of "Uncle Dick."

The Hercules, one of the heaviest and best of Fairleigh's on the Iquique Railroad at that time, had the following dimensions:

Cylinders, 4 in number	15″ × 22″	Heating surface, fire box	138□′
Driving wheels, 12 do.	42″	" " tubes	1 543□′
Internal diam. of boilers	45″	Total heating surface	1 681□′
Length of each barrel	9′ 9″	Steam pressure in boilers	130 lbs.
Length of fire box	7′ 6″	Total weight empty	44 tons.
Width "	3′	" " with boiler filled	48 "
Height "	4′ 9″	Water carried in tanks (2 200 imp. gallons)	10 "
Number of tubes	286	Fuel in bunkers	2 "
Int. diameter of tubes	$1\frac{3}{4}$″	Total weight of loaded engine	60 "
Length of tubes	11′ 1″		
Area of fire-grate	23□′		

Speaking from remembrance only, the above steam pressure was frequently much exceeded, to enable the engine to pull over difficult places.

As the engine first drew her supply of feed water from tank cars in the train, which were left on the way at sidings as emptied, its actual weight over the worst portion of the road must have been nearly her full loaded weight—certainly as much as 58 tons; and, like most of Fairleigh's engines, the entire weight was upon the 12 drivers. (The above weights and dimensions being from English sources, the tons are, doubtless, gross ones.)

The most difficult portion of the road below Sta. Rosa, where the engines were frequently stalled, and compelled to stop for some time to get up steam, is a 4 per cent. grade upon a reverse curve, the sharper curve being, I think, about 16° (not sharper than that), and more than long enough to receive an upward bound loaded train.

Assuming, with the author, an equalization of 1° in curvature with 0.05 foot grade per 100 feet—which, being equal to 1 lb. per net ton traction per degree of curvature, is a sufficiently liberal allowance, to say the least, on these sharp curves, in view of the experiments upon the Metropolitan Railroad referred to by Mr. Chanute (Trans. Am. Soc. of Civ. Eng., Vol. VII, p. 98,)—the work of the engine would be only that of hauling its train up $4\frac{8}{10}$ per cent. grade on a tangent.

Immediately above Sta. Rosa, the reverse curves above described, probably render the ascent more difficult from their length so reducing

steam pressure in the boilers, as to make it exceedingly difficult for a heavily loaded engine to pull through, even with the benefit of the speed obtained by a run upon the level track at the station, although the grade is no steeper nor the curves much sharper than at the most difficult point below Santa Rosa. It is probably equivalent to a $4\frac{9}{10}$ per cent. grade on a tangent.

As stated by the English superintendent of motive power and machinery at that time, the Hercules had taken up 150 tons (gross), but he considered this amount too much for the engine, and that 120 tons (gross) was a fair working load.

My own opinion, from frequent journeys over the road—although from the very promiscuous loading of the flats, it was difficult to judge of the amount of freight on each car—is that the latter estimate is quite a liberal one. I give as a sample the minutes of one trip, premising that all the cars were open flats, that the English ones were of ordinary English make on 4 wheels only, that the iron tank cars, also on 4 wheels, carried each about $10\frac{1}{2}$ net tons of water, that "Engineers' Camp" siding is a short distance above the Santa Rosa curves, and that from thence to the terminus at La Noria, the grades are moderate.

"Hercules left Iquique a little before 10 A.M., with three tank cars only; took on 3 loaded American and 1 loaded English car at the reversing station. Arrived at Molle ($9\frac{1}{2}$ miles) at 11 A.M.; took one more loaded American car, and left one tank after emptying it; two hours spent at Molle in transferring water, and *getting up steam.* Reached Engineers Camp (19 miles from Iquique) at 3 P.M.; 3 loaded English, 7 empty English, and 3 empty American cars added to the train, and one more tank left behind. Train arrived at La Noria ($33\frac{1}{2}$ miles from Iquique) at 6 P.M. Water being nearly exhausted (8 500 gallons taken up exclusive of boiler charge) engine returned without load."

Other engines of the same make were ordered, during the time spoken of, with 17×22 inch cylinders, 45 inch drivers, and 2 059 square feet heating surface, but I have obtained no record of their performance.

EDWARD P. NORTH.—In the fall of 1868 I was directed to locate a line for a temporary track, eight miles long, around some heavy work at the head of Echo Canon, Union Pacific Railroad.

It was found advantageous to employ a grade of $145\frac{1}{2}$ feet per mile and two Y's in getting down into the valley of the creek that was followed. The stems of the Y's, which were long enough to hold seven

freight cars and a locomotive, had a grade of 145½ feet per mile against any train that came on them, and were extended in sharp vertical curves. At the foot of the heavy gradient, grading was done for a safety spring switch like the one described by Mr. Flagg, but track was never laid on it, as no difficulty was found in controlling the small trains that the Y's would hold. As far as known by the writer, this was the first instance in which such vertical curves were used.